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UPLIFT IN MASONRY DAMS

Final Report of the Subcommittee on Uplift in Masonry Dams of the Committee on Masonry Dams, 1951

POWER DIVISION

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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REPORTS

UPLIFT IN MASONRY DAMS

FINAL REPORT OF THE SUBCOMMITTEE ON UPLIFT IN MASONRY DAMS OF THE COMMITTEE ON MASONRY DAMS OF THE POWER DIVISION, 1951

Scope of Report

Herewith is submitted a report by the Subcommittee on Uplift in Masonry Dams. The Subcommittee has been functioning for several years under an instruction "to study the possibility of clarifying the uplift problem."

The available information has been assembled and studied. The data applicable to the question of foundation uplift are held to be adequate to warrant a report on this phase. There are numerous unanswered questions and a scarcity of field data as to uplift effects in the concrete of the dam structure, and prospects for obtaining such information in the near future are not very promising. Hence, this report is presented as a statement on foundation uplift. Since the dimensions of a dam depend on the assumptions for foundation uplift, these are of primary importance.

Assumptions upon which any dam is designed are the responsibility of the designer and cannot be waived by him. Therefore, neither this Subcommittee nor anyone else can arrive at rules for such assumptions of general or universal applicability. A few conclusions will be presented at the end of this report, but they must not be considered as rules. The report is essentially a digest of information and practice which, when considered in connection with the particular circumstances of each project, should aid the designer in arriving at adequate assumptions. Also this report should not be considered as establishing any rules by which the adequacy of existing dams should be determined. Here again, every existing structure is a special case, and any determination of its adequacy is a matter for individual expert study.

The uplift question is only one of a number of factors entering into the design of masonry dams. Others are ice pressure, assigned to another ASCE committee; earthquake effects, temperature effects, and other factors causing volume change; and the specification of materials and processes to secure concrete of adequate quality as to strength and permeability. The task of this Subcommittee has been to study uplift, and others presumably will investigate additional phases.

1. Failures of Dams

The most noteworthy failures of concrete or masonry gravity dams in the United States have been those of Austin, Tex.; Austin, Pa.; the St. Francis Dam in California; and Dam No. 26 on the Ohio River. In all these cases failure was due to inadequate resistance in the foundation to horizontal movement—that is, to shearing failure in the foundation. Also, in all these cases there was evidence that uplift contributed to the failure by reducing the effective weight of the structure, and, consequently, the frictional resistances dependent upon weight.

At least in the United States, no important failure has been due to overturning of a dam; nor have any failures been ascribed to the development of internal pore pressures in the concrete or masonry itself.

These considerations lead to the conclusion that uplift effects, especially in the foundation, are always to be reckoned with and are an essential element of design.

2. Existing Dams

While modern practice has come to include an uplift allowance, many existing structures, designed without any allowance for uplift—perhaps without any conception of it—have been for many years, and are still, functioning satisfactorily. Similar attitudes have existed with reference to design against ice pressures. Some of these are in the category of high structures, although most of them are probably less than 100 ft in height and may be classed as low dams.

Among the reasons which may be assigned for the apparent stability of these structures, even though no allowance for uplift forces entered into their design, may be listed the following:

a. The presence of joints or other openings in the foundation which facilitate downstream flow in the rock and discourage the building up of any great uplift intensity. Coupled with this is possible sedimentation or other means of sealing the entrance to water passages upstream of the dam which would still lower the pressure gradient further. The practice of providing a cutoff wall through the upper strata of the foundation, whatever may be its disadvantages from other points of view, has had at times beneficial effects upon the uplift intensity. The principal objection to the use of a cutoff wall in rock, aside from its cost, appears to be that the necessary blasting will open passages or joints which otherwise would remain closed and thus facilitate the entrance of water.

b. While tensile strength in the foundation and in the concrete or masonry, as well as in the connecting bond, is generally neglected in design, such strength does exist. In dams of moderate height, this factor may have great significance. Particularly when the foundation rock is of massive, and steeply dipping character, tensile and shearing strength may provide material anchoring forces at the "heel" of the dam, directly contributing to stability. In horizontally stratified and thinly bedded rocks such resistances are less significant. In any case, these effects become less important as the height of the dam increases. While the destructive forces increase as the square of the height of the dam, the resistance due to tensile strength increases much less rapidly. Tensile strength will

help to account for the performance of many light dams of considerable age which were built on strong foundations.

c. While dams are customarily designed upon the "middle third" theory, assuming a straight-line distribution of the load stress across the foundation, there is actually some uncertainty about this distribution. Some mathematical analysis and some model tests have shown departures from the straight-line distribution.[1]

An illuminating illustration of departure from the straight line is found in the stress distribution determined for Shasta Dam in California.[2] In this case stresses much less than the maximum were determined at the toe of the dam, even at a time when the reservoir was almost full. Temperature effects were moderate. While this determination was made upon a very large dam, there seems to be no reason why material departure from the straight-line assumption may not exist in much smaller structures.

d. Related to Section 2*c* are the effects of temperature load transfer and swelling of masonry at the face exposed to water, which have the result of modifying the pressure gradient from a straight line and increasing the intensity of pressure at the heel and toe of the dam above that given by the straight-line distribution. These effects may not be significant in low dams, where tensile strength is relatively important, but they can be important in high structures.[3,4]

e. The resistance to horizontal movement as expressed by the shear friction factor is actually much greater than any resistance related only to frictional factors, such as the sliding factor.

f. In some sites there are peculiarities favorable to stability, such as the shape of abutments in narrow canyons. In such cases arch effects may be realized which offset any deficiency in a gravity section.

3. Uplift Pressure

There are two constituent elements in uplift pressure—the area factor and the intensity factor.

The area factor, A, is the proportion of a horizontal section at the base of the dam which is assumed to be subject to uplift pressure. It has a maximum value of 100%, or 1. Smaller values have been assumed in numerous dams, according to the judgment and experience of the designer.

The intensity factor, I, is the assumed ratio which intensity bears to an intensity gradient extending from maximum headwater to maximum tailwater. It is commonly expressed as a gradient extending from maximum tailwater to some assumed higher elevation at the upstream face of the dam, below the maximum headwater elevation. Thus, a 50% intensity factor indicates a gradient, the upstream end of which is at an elevation midway between maximum headwater and maximum tailwater.

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[1] "Stress Studies for Boulder Dam," *Bulletin No. 4*, Part V, Boulder Canyon Project Final Repts., U. S. Bureau of Reclamation, U. S. Govt. Printing Office, Washington, D. C.

[2] "The Development of Stresses in Shasta Dam," by J. M. Raphael, *Proceedings-Separate No. 116*, February, 1951.

[3] "Measurements of the Structural Behavior of Norris and Hiwassee Dams," *Technical Monograph No. 67*, TVA, Knoxville, Tenn., 1951.

[4] "Hiwassee Project Report," Part I, TVA, U. S. Govt. Printing Office, Washington, D. C., 1948.

In Fig. 1 is shown a break in the intensity diagram at the line of drains if such are used. The diagram with a "break" conforms more nearly to observed data as cited later whereas the straight-line assumption simplifies computation somewhat. In either case the intensity factor expresses the relation of the intensity assumed to that which would occur if the gradient extended between maximum headwater and tailwater in a straight line. An intensity factor of 100% or 1 would mean that the assumed gradient extends from maximum headwater to maximum tailwater.

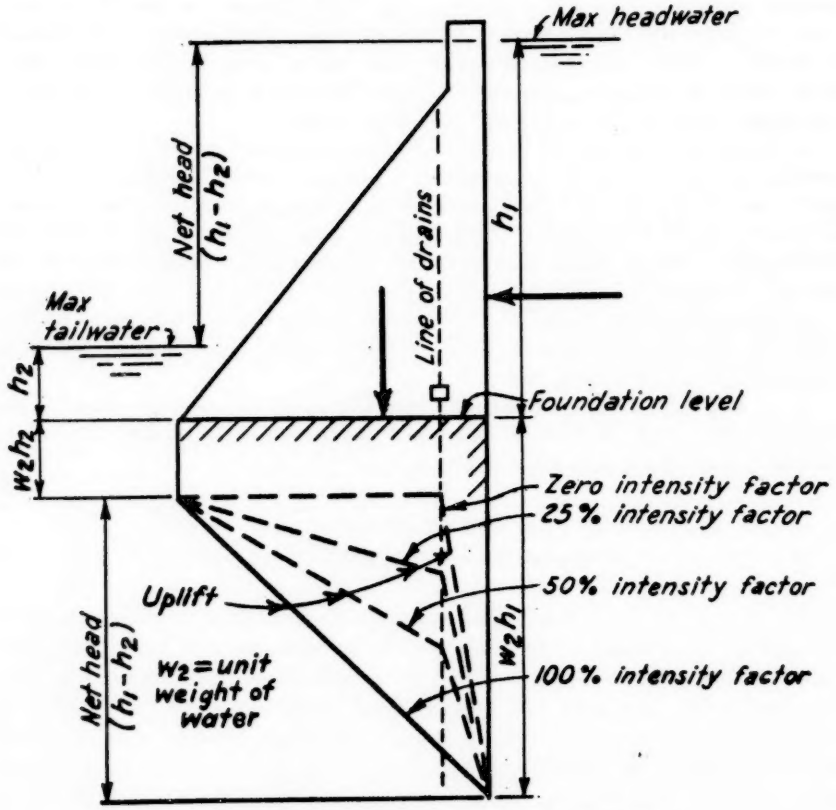


FIG. 1.—DIAGRAM OF UPLIFT INTENSITY WITH VARIOUS FACTORS

There has been some confusion in the past as to the use of these factors. They are not interchangeable. In a high dam with low tailwater it does not make much difference whether, for example, I be taken as 1.0 and A as 0.50, or I be assumed as 0.50 and A as 1.0; but in a low dam with high tailwater such interchange makes a great difference in the computed uplift effect.[5] It is

[5] "Engineering for Dams," by W. P. Creager, J. D. Justin, and Julian Hinds, John Wiley & Sons, Inc., New York, N. Y., 1945, Vol. II, p. 269.

sought in this report to adhere to the definitions previously stated, and to express all the ideas presented in these terms.

This report is considering primarily the matter of foundation uplift. In order to secure consistency of treatment through the entire length of the structure—that is, in the abutments as well as in the maximum section—the same design assumption should be made for all levels in the dam above the foundation. Modifications may be made for differences in tailwater conditions which would lead to steeper surface slopes on abutment sections, but designers will generally prefer to use uniform slopes in order to minimize construction difficulties and to secure satisfactory appearance.

4. Importance of Exploration and Geological Study

The proper determination of these factors, especially I, must be based on the most thorough possible study of the foundation and geological conditions. It should include determination of the geologic structure and its significance, the composition, absorptive capacity, and permeability of the rocks. The prevailing bedding and joint systems; the thickness and relative tightness of strata; and the presence, magnitude, and course of solution passages and of any other systematic or unsystematic openings are significant. The ground-water table as it exists, and as it may be expected to be modified by the proposed construction, is important. If there is evidence of underground flow in a direction parallel to the dam axis, the proper uplift assumption may be affected. The quality of water to be impounded should be studied—for instance, whether it is hard or soft—to determine the possible effect of the water on drains and grouted cutoffs. In short, it is not possible to know too much about subsurface conditions.[6,7] (See case history of Center Hill Dam in Section 7.) Tables 1(a), 1(b), and 1(c) present considerable condensed data on foundation conditions, but individual sites require much more detailed investigation than these tables indicate.

5. Evidence as to Area Factor

The value of the area factor, A, in foundations or in structures has not been determinable *in situ* by any known method. There is some experimental evidence believed to be pertinent, and there are certain arguments and theoretical discussions by authorities which constitute additional evidence.

The most recent experiments upon the effective area in concrete include those of Karl Terzaghi, Hon. M. ASCE, and S. Leliavsky Bey and Douglas McHenry, Members, ASCE. The first investigation, by Mr. Terzaghi, published in 1936, reached a value of 0.98 as the area factor in concrete.[8] Leliavsky Bey, after an elaborate series of experiments, obtained a mean value of 0.91 from his work on concrete.[9] When the concrete was considered to be in a saturated condition, he corrected this value to 0.85. Mr. McHenry, reported to the Third Large Dams Congress in 1948 a value of 1.00± for the area

[6] "More and Better Geological Data for Use in Hydraulic Projects," by G. H. Matthes, published by the Am. Inst. of Mining and Metallurgical Engrs., 1929.

[7] "Geology and Foundation Treatment for TVA Structures," TVA, U. S. Govt. Printing Office, Washington, D. C., 1949.

[8] *Engineering News-Record*, June 18, 1936, p. 872.

[9] "Experiments on Effective Uplift Area in Gravity Dams," by Serge Leliavsky Bey, in "Uplift Pressure In and Beneath Dams: A Symposium," *Transactions*, ASCE, Vol. 112, 1947, p. 444.

TABLE 1.—FOUNDATION DATA,

Line No.	Name of dam (1)	Height of dam (ft) (2)	Type of rock (3)	Variety of rock (4)	Bedding thickness (5)
					(a) BUREAU OF RECLAMATON
1	American Falls	...	Igneous	Columnar basalt	Solid and massive
2	Hoover	...	Metamorphic	Andesitic tuff breccia	Massive
3	Friant	...	Metamorphic	Quartz-biotite schist	Massive
4	Grand Coulee	...	Igneous	Fine-grained granite	Varies with shearing
5	Gibson	...	Sedimentary	Crystalline-limestone	Hard massive
6	Marshall Ford	...	Sedimentary	Austin and glenrose limestone	Thin
7	Owyhee	...	Igneous	Porphyrite or glassy rhyolite	Hard and durable
8	Seminoe	...	Igneous	Dark gray granite[d]	Blocky, with thin mica seams
					(b) TENNESSEE VALLEY AUTHORITY
9	Wheeler	72	Sedimentary	Argillaceous limestone	Thin, 3 in. to 6 in.
10	Fontana	480	Metamorphic	Quartzite[e]	Massive
11	Douglas	202	Sedimentary	Dolomite	Massive
12	Cherokee	173	Sedimentary	Shale and limestone	Thin
13	Hiwassee	307	Metamorphic	Schist and quartzite	Variable
					(c) CORPS OF ENGINEERS
14	Clark Hill	200	Igneous[g]	Granite or granite gneiss	Varies (thin to massive)
15	Allatoona	190	Metamorphic	Schist, quartzite, dolomite	Schist foliated; quartzite massive
16	Buggs Island	144	Igneous[g]	Granite and biotite gneiss	None
17	Philpott	220	Metamorphic	Schist	Foliated
18	Tygart	207	Sedimentary	Spillway sandstone	Thin to medium
19	Bluestone	165	Sedimentary	Abutment shale	Thin to massive
20	Conemaugh	137	Sedimentary	Sandstone, shale, fire-clay, etc.[k]	Thin to medium
21	Wolf Creek	240	Sedimentary	Limestone	Thin to massive
22	Center Hill	240	Sedimentary	Limestone	Thin to medium
23	Dale Hollow	185	Sedimentary	Limestone	Thin to massive
24	Stewarts Ferry	145	Sedimentary	Limestone with shale intercalations	Thin to massive
25	Narrows	190	Sedimentary	Sandstone and shale	Thin bedded
26	Norfork	220	Sedimentary	Dolomitic limestone	Thin to medium
27	Bull Shoals	258	Sedimentary	Dolomitic limestone	Thin to medium
28	Fort Gibson	110	Sedimentary	Limestone with shale intercalations	Thin to medium bedded
29	Fall River	66	Sedimentary	Shale, sandstone, and limestone[m]	Thin to medium
30	Harlan County	106	Sedimentary	Chalk with bentonite intercalations	Thin to medium
31	Conchas	235	Sedimentary	Shale and sandstone	Thin to medium
32	Caddoa	130	Sedimentary	Sandstone	Thin to massive
33	Whitney	155	Sedimentary	Limestone	Thin to medium
34	Chief Joseph	220	Igneous	Granodiorite	Master joint system in granite[r]
35	McNary	158	Volcanic	Basalt	Massive
36	Bonneville	170	Igneous[t]	Diabase (lock and power house)	Medium to massive tuff (main dam)
37	Detroit	390	Volcanic[u]	Andesite and breccia diorite	Medium to massive
38	Meridian[w]	250	Volcanic[t]	Andesite tuff breccias	Massive flows
39	Cottage Grove	95	Pyroclastic	Tuffs	Medium
40	Dorena	145	Volcanic[t]	Tuffs, agglomerates, andesite flows	Flows of andesites[z]
41	Folsom	268	Igneous[g]	Schists and metabasite[aa]	Foliation[bb]
42	Pine Flat	440	Metamorphic	Amphibelite	Debatable[dd]

[a] With reference to the axis of the dam. [b] At the maximum section. [c] Between centers. [d] Broken by seams and fracture. [e] Phyllite in left abutment. [f] Maximum depth. [g] Igneous and metamorphic. [k] Local solution in dolomite. [t] Each of the dams in Table 1(c) has a drainage gallery and, with two excep-

Dams in the United States

Line No.	Dip and Strike Angle[a]		Jointing (spacing and tightness)
	Dip (6)	Strike (7)	(8)
	(USBR) Dams		
1	Upstream about 90 ft per mile	...	Tight
2	Downstream	North and South	In many directions
3	Downstream	N 55° W	Few and tight
4	Irregular in direction		Average
5	West 70° to 86°	N 5° W	Numerous seams and joints
6	Northward 5 ft in 1,000 ft.	...	Numerous seams and parting planes
7	Westward 5° to 10°	...	Visibly fissured
8	Irregular in direction		Tight
	(TVA) Dams		
9	Horizontal		Moderate
10	Downstream 10°	Transverse to river, 30°	Frequent
11	Transverse to river, 10° to 25°		Moderate
12	Downstream 15° to 45°	...	Frequent
13	Transverse to river, 50° to 80°		Frequent
	(USED) Dams		
14	Upstream 45° (varies)	Almost parallel to axis	Wide, tight
15	Upstream 40° to 70°	Parallel to axis	Wide, mostly tight[h]
16	Not bedded		Most wide, tight
17	Downstream 40° to 60°	Almost parallel to axis	Wide, mostly tight
18	5° into left abutment	Almost normal to axis	Medium to wide, tight
19	Nearly horizontal		Close and open[j]
20	Nearly horizontal		Wide and tight
21	Nearly horizontal		Wide and tight, some channeling[l]
22	Slight dip to right abutment	...	Medium to wide, tight
23	Downstream slightly	Almost parallel to axis	Tight to open, some channeling[l]
24	5° toward right abutment	45° to axis	Wide, numerous open and weathered
25	70° downstream	11° to axis	Close jointed, tight
26	Nearly horizontal		Medium to wide, some channeling[l]
27	Nearly horizontal		Medium to wide, solution channels
28	Nearly horizontal		Wide and tight, some channeling[l]
29	Nearly horizontal		Wide and tight
30	Nearly horizontal		Wide and tight
31	Nearly horizontal		Wide and tight
32	Nearly horizontal		Medium and wide
33	Nearly horizontal		Wide and tight
34			Close to medium[s]
35	Horizontal flows		Wide and tight
36	Nearly horizontal		Wide, tight to open
37	Nearly horizontal		Variably open[v]
38	Steep downstream[z]	Diagonal to axis	Wide and tight[y]
39	East 7°	Diagonal to axis	5 ft to 20 ft apart, open
40	20° to 30° upstream	Parallel to axis	Columnar and platy[r]
41	Variable foliation in Mormon Dam, diagonal to axis		Close to medium, weathering[ee]
42	Upstream 40° to 70°[ee]	...	Close to medium, mostly tight

tions, the curtain grouting is done from the gallery. At Tygart Dam (line 18) and at Conchas Dam (line 31), the grout curtain was placed through the heel of the dam. [j] A fault at the right end of the spillway. [k] Also thin limestone and coal. [l] Solution channeling. [m] Interbedded. [n] Across fault zone only. [o] On

TABLE 1.—(*Continued*)

Line No.	Name of dam	Compressive strength (lb per sq in.) (9)	Depth of grout curtain[b] (ft) (10)	Depth of drainage (ft) (11)	Spacing of drains[c] (ft) (12)
		(*a*) Bureau of Reclamation (USBR) Dams			
1	American Falls	Not available	20	20	5
2	Hoover	15,000	480	30 to 50	20
3	Friant	High	100	50	20
4	Grand Coulee	High	300	50	20
5	Gibson	Not available	40	40	4 to 7
6	Marshall Ford	Not available	125	40	20
7	Owyhee	400	270	110	10 to 20
8	Seminoe	Not available	100	100	5 to 10
		(*b*) Tennessee Valley Authority (TVA) Dams			
9	Wheeler	High	30	...	...
10	Fontana	High	150	50	10
11	Douglas	High	200[f]	40	8
12	Cherokee	Low	100	40	8
13	Hiwassee	Variable	80	40	8
		(*c*) Corps of Engineers (USED) Dams			
14	Clark Hill	...	80	30	12
15	Allatoona	1,900 to 44,000	75	20	5
16	Buggs Island	6,721 to 19,118	50	40	10
17	Philpott	3,210 to 7,018	100	To be determined	...
18	Tygart	200 to 5,000	160[i]	100	2.5
19	Bluestone	1,095 to 8,810	150	100	10
20	Conemaugh	1,095 to 8,810	To be determined	...	...
21	Wolf Creek	2,840 to 13,100	75	65	10
22	Center Hill	6,180 to 13,860	100	80	10
23	Dale Hollow	3,110 to 9,640	100	50	10
24	Stewarts Ferry	6,132 to 7,050	...	...	...
25	Narrows	913 to 34,900	100	40	10
26	Norfork	2,100 to 30,200	150	53	8
27	Bull Shoals	25,000 average	200	60	8 and 9
28	Fort Gibson	813 to 13,200	75	40	10
29	Fall River	70 to 5,000	30	20	10
30	Harlan County	1,227 to 1,400	85[n]	50 to 75[o]	5 and 10[p]
31	Conchas	1,067 to 9,717	56[q]	84	15
32	Caddoa	2,510 to 6,150	52	35	10
33	Whitney	4,000	150	60	7
34	Chief Joseph	12,288 to 34,960	To be determined	To be determined	7
35	McNary	3,900 to 44,600	Varies from 60 to 100	50	10
36	Bonneville	370 to 1,250	55	50	60
37	Detroit	6,000 to 48,000	150	To be determined	To be determined
38	Meridian[w]	4,050 to 7,250	150	100	10
39	Cottage Grove	1,100 to 4,700	50	50	20
40	Dorena	None available	75	40	10
41	Folsom	Not available	50	40	10
42	Pine Flat	9,350 to 31,500	150	To be determined	10

the upstream side alternating between 50 ft and 75 ft; downstream depth, 45 ft. [p] On the upstream side of the grout gallery, 5 ft and 10 ft on the downstream side. [q] The grout curtain for the main dam was placed through the fillet at the heel of the dam. Provision for future grouting was made by constructing a grouting gallery along the axis of the main dam approximately 15 ft above the foundation. This gallery was provided with 3½-in. pipe casings set on 5-ft centers and extending from foundation rock to the gallery floor. [r] Close and tight. [s] Mostly tighty with some weathering. [t] Pyroelastic. [u] With igneous dikes. [v] Variably open and weathered abutment, tight in valley bottom. [w] Lookout Point. [x] Flows tilted 5° to 20° into right abutment. [y] Fault to the left of the concrete section. [z] Over fragmentary beds of tuffs, agglomerate. [aa] Main dam to Mormon Island Dam. [bb] Foliation in Mormon Dam, none in the main dam. [cc] Close to medium and extensive weathering from 20 ft to 30 ft, moderate weathering to 105 ft. [dd] Upstream dip. Joints most pronounced. [ee] Most pronounced dip, secondary dip is set normal to these.

factor,[10] after experiments on concrete with the triaxial compression machine at Denver, Colo. The mean of these values is 0.94.

As to theoretical evidence, L. F. Harza,[11] M. ASCE, has presented an argument by deductive reasoning that the area factor must be 1.0. His reasoning is applied primarily to concrete, but it appears to be equally applicable to rocks in which there is defined porosity, and this includes a large number of such rocks. The essence of this paper was also presented to the Third Congress on Large Dams at Stockholm, Sweden. It has been widely discussed.

In a previously cited work[5] a strong argument is made for the use of an area factor of 1.0 in foundation rocks, particularly when these are horizontally bedded. Mr. Terzaghi argues[12] that conditions similar to those which he found in concrete can be expected in rocks which are measurably porous, and that these rocks in their own structure will be subject to a high area factor. This is aside from the influence of defined openings in the rock, whether they be joints, openings along beds, or the results of solution. The presence of these defined openings facilitates access to rock which is porous and promotes the saturation thereof and the development of a high area factor. The defined openings may themselves occur in such a position as to present an area factor of approximately 1.0.

In 1933 W. A. Zisman experimented on the compressibility of rocks by applying hydraulic pressures to jacketed and unjacketed specimens by a method similar to that followed by Mr. Terzaghi in his experiments on concrete.[13] The compressibility of most of the rocks examined was markedly greater in the jacketed specimens than in the unjacketed ones. These results point to a high value of the area factor in these rocks. The Chairman of this Subcommittee has estimated from the given data that a specimen of marble had an area factor of 0.90, while specimens of three different granites had a factor of approximately 0.75. An exception was found in the case of obsidian, which, because of its glassy nature, had no porosity and showed no difference between the performance of jacketed and unjacketed specimens. This confirms Fig. 2 as mentioned later. Mr. Zisman's data also indicate that, when the pressure intensity reached 720 kg per sq cm (10,200 lb per sq in.), the difference in compressibility between unjacketed and jacketed specimens very markedly diminished, indicating that pores and cracks were closed by the high pressure.

In the discussion and closure of Mr. Harza's paper, previously cited, it is pointed out that the existence of pore pressure area up to values of 100% can occur together with a concurrent high degree of structural strength which will resist the disruptive effects of pore pressures. Similarly, the body of unweathered foundation rock will be characterized usually by even higher qualities of structural strength. However, if weathered rock exists in the foundation, as it frequently does along joints, the weathered material can

[10] Discussion by Ross M. Riegel of "The Significance of Pore Pressure in Hydraulic Structures," by L. F. Harza, *Transactions*, ASCE, Vol. 114, 1949, p. 250.

[11] "The Significance of Pore Pressure in Hydraulic Structures," by L. F. Harza, *ibid.*, p. 193.

[12] *Proceedings*, ASTM, Vol. 45, 1945, p. 777.

[13] *Proceedings of National Academy of Sciences*, Vol. 19, 1933, p. 666.

be characterized by a high area factor with little or no structural strength. Whereas weathered material is usually removed before construction, the possibility that some residuals will still remain in the foundation is strong and must be recognized.

An interesting statement was made by W. Schurter, general reporter of the Third Large Dams Congress at Stockholm, in the report of that body, namely:

> "The ratio between the area effectively submitted to uplift pressure and the total area considered is an essentially empirical value, which may be determined only by experiments.
>
> "Investigations, communicated to the Third Large Dams Congress by different authors, have supplied for this coefficient, values included between 0.85 and 1.0.
>
> "If special steps are not taken against uplift pressure, the action of the aforesaid is, in principle, to be felt in the substratum of the foundation area and in the body of the dam."

In the same congress, E. Reinius stated: "In Sweden the effective uplift area is always assumed to be 100 percent of the total area."

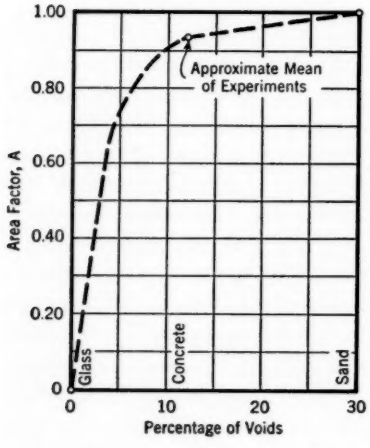


FIG. 2.—AREA FACTOR RELATED TO VOIDS, ACCORDING TO W. P. CREAGER

An interesting diagram (Fig. 2) has been submitted by W. P. Creager, Hon. M. ASCE, of this Subcommittee. Based on an admission of 1.0 as the area factor in sand, and a factor of 0 in an absolutely nonporous material such as glass, the area factor would vary roughly as shown in the diagram, giving the general position of experimental values.

The further fact may be cited that near the base of high dams, where tailwater is normally present and may have a high value, a condition of ultimate saturation must be regarded as attainable. With it is a tendency to a high area factor.

In the light of this experimental evidence and argument, most of the members of the Subcommittee believe that as a practical matter all new dams should generally be designed with an assumed area factor of 1.0.

6. METHODS OF REDUCING INTENSITY FACTOR *I*

The first method of reducing *I* is that of the grouted cutoff. The object, of course, is the prevention or retardation of the passage of water beneath the dam and the production of a reduced uplift. The depth, frequency of holes, and pressure employed will be determined by the geological conditions, as revealed by the exploration. A great deal of trial drilling and grouting is generally advisable before a final program is arrived at. Useful preliminary

information may be obtained from the accounts of similar operations elsewhere and from Tables 1(*a*), 1(*b*), and 1(*c*). Wherever a definitely impervious stratum exists in the foundation, it is usually advantageous to connect the grout curtain with this stratum. (However, see discussion of Center Hill Dam in Section 7.) Where definitely pervious strata are known to exist the curtain should cut off these formations. Where there is little variation in the strata, approximate flow nets may be utilized to determine the desirable depth of cutoff. It will be seen that no rule can be given for this depth.

The grouted cutoff is regarded by most engineers as permanent. However, at the Third Large Dams Congress, Mr. Reinius of Sweden expressed some doubt as to this permanency. In any event if a gallery is provided near the foundations, as is usual in the larger American dams, additional grouting of the cutoff can be performed from this gallery if it is ever required. In the case of lower dams, such grouting can be performed either initially or later through the water of the reservoir. Evidence as to the advisability of later treatment would be supplied by observation of leakage.

In addition to the grouted cutoff, an additional provision in many American dams is a system of drainage holes which is drilled from a gallery in the dam near its upstream face to penetrate the foundation rock downstream of the cutoff so that water passing the cutoff area can escape to the gallery without building up the uplift intensity. This water escapes to the downstream face or to tailwater.

The relief will be more effective and reliable with closer spacing of the drain holes, although it is always possible that the drains will not reach certain passages. Data on measured uplift pressure and on spacing and depth of drain holes are furnished in Section 7. Certain requirements as to drainage wells must be stated. The gallery from which the holes are drilled should be large enough to permit the use of machinery for the cleaning, reaming, or extension of holes or for drilling additional holes, and the holes should be so located in the gallery as to permit such operations.

There is a real objection to drain holes in that they may become clogged and made ineffective by deposits. Regular inspection of the condition of the drains is an essential condition of their use, and cleaning, scraping, or reaming must be anticipated when necessary. The drilling of a new set of drains may possibly be necessary if other measures are ineffective. Several instances have come to the attention of the Subcommittee where measured uplift pressures have been reduced by drilling additional drain holes, notably at Hoover Dam (in Arizona and Nevada) and Norris Dam (in Tennessee).[3,14]

The clogging of drains may arise from the following causes:

a. Evaporation, which may at times leave deposits of calcium carbonate or other minerals upon the exposed surface of the drain hole.
b. Reduction in the capacity of the water to carry solids because of its loss of pressure on escaping into the drains.
c. Increase of alkalinity by the solution of salts contained in the rocks.
d. Collection of trash or debris in the drains.

[14] "Uplift Pressures in Concrete Dams," by Kenneth B. Keener, *Transactions*, ASCE, Vol. 16, 1951, p. 1218.

Swedish engineers are opposed to the use of drains because of troubles experienced in their country. On the other hand, Tables 1(*a*), 1(*b*), and 1(*c*) show a wide use of drains in the United States, and the Subcommittee has not been informed of troubles similar to those reported in Sweden. It is suggested that Swedish waters in general have much lower alkalinity than those of the United States and, therefore, are much more active in their effects upon concrete and rocks with soluble elements. As suggested in Section 4, the quality of reservoir water is one of the factors requiring examination in the preliminary investigations.

The most satisfactory check on the performance of the drains is furnished by observations of the uplift pressure occurring under the structure (supplemented by measurements of the actual flow), and these observations are most permanently reliable when made from transverse galleries in the dam. In structures of any size when reliance is placed on the effectiveness of the drains, the provision for uplift measurement should be an essential feature of the design.

Members of the Subcommittee have called attention to the wide buttress-type dam constructed in Sweden and Scotland. In this type drains can readily be installed between the buttresses and a very complete drainage of the entire foundation can be secured. The same result can be obtained by a system of galleries transverse to the axes of solid dams with drains drilled in the floor thereof. Subsurface conditions may make such drainage advisable. The selection of a type of structure is not, however, the function of this Subcommittee, and is a question of economy of design. It is held to be the Subcommittee's function to deal generally with so-called gravity dams.

7. Evidence As to Intensity Factor in Foundations

Herewith is summarized the available data on intensity of actual foundation uplift as measured in the structures themselves.

a. The first publication which may be classed as giving modern information on the subject is that of Julian Hinds,[15] M. ASCE. (The data given on two masonry dams, Willwood (in Wyoming) and American Falls (in Idaho), are included in the following paragraph.)

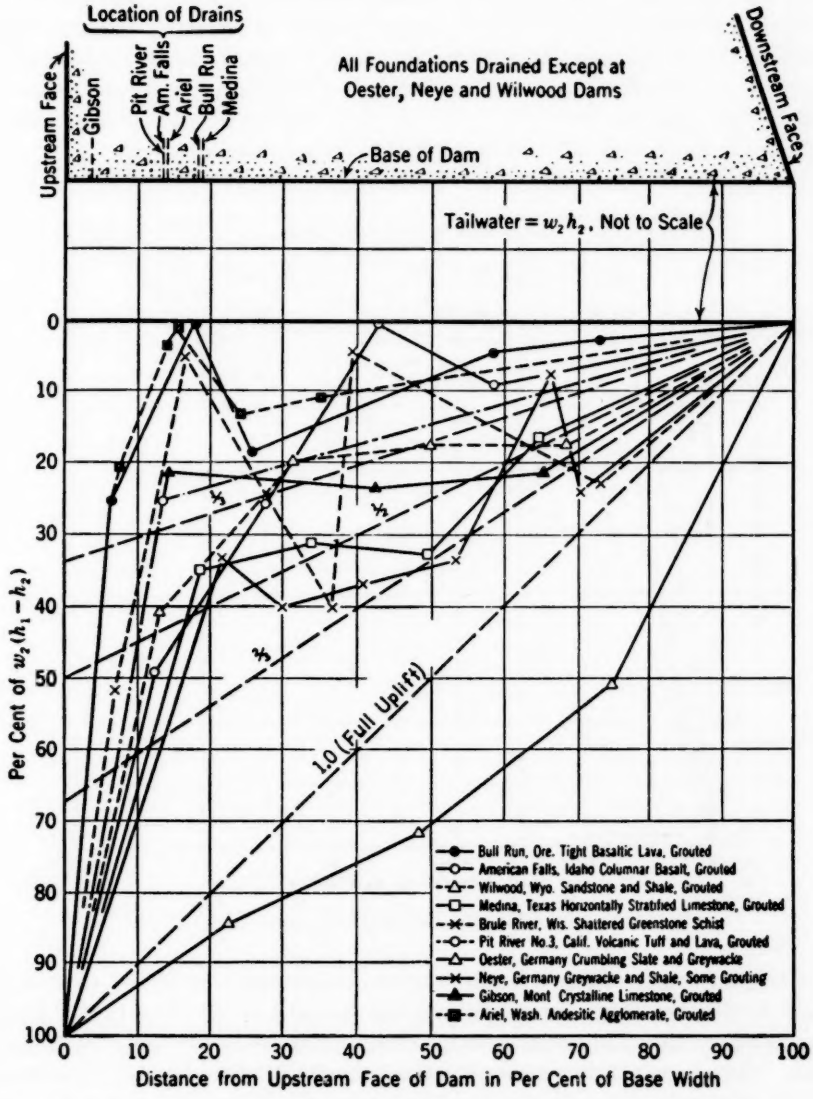
b. Another reference is to the collection made by Ivan E. Houk,[5,16] M. ASCE, and reproduced as Fig. 3. In this diagram are shown measured intensity diagrams for structures with data as to cutoff grouting and drainage provisions. It is to be noted that the highest intensity is shown for a structure having neither grouting nor drainage and that those structures in which both grouting and drainage have been provided show the lowest range of intensities.

c. In Fig. 4 are shown separately data for Neye (in Germany) and Willwood dams listed by Mr. Houk, for which cutoff grouting was performed but no drainage was provided. To these have been added a record from Wheeler Dam of the Tennessee Valley Authority (TVA) compiled by the Chairman of

[15] "Upward Pressure Under Dams," by Julian Hinds, *Transactions*, ASCE, Vol. 93, 1929, p. 1527.

[16] "Measurements of Existing Structures," by Ivan E. Houk, *Civil Engineering*, September 1932, p. 578.

this Subcommittee and hitherto unpublished. The record of Wheeler Dam (in Alabama) is a composite of readings at nine stations over a period of ten years. The foundation characteristics at Wheeler are given in Table 1(*b*).



Eig 3.—Maximum Observed Hydrostatic Pressure Under Existing Gravity Dams on Rock Foundations

This dam is a relatively low structure, being subject to only about 50 ft of net head. In these dams there is a marked reduction from 100% intensity as previously defined. For convenience the average line of the Bureau of

Reclamation (USBR), United States Department of the Interior (from Fig. 9), is included in this diagram.

There is much more information available as to dams which are provided with both grouting and drainage, particularly from the USBR and the TVA.

d. Information for eight dams of the Bureau of Reclamation has been given by K. B. Keener, M. ASCE, in a paper before the Society's Power Division in November, 1949, at Washington,[14] from which Fig. 5 is reproduced. Foundation data for these dams, furnished by Mr. Keener, are presented also in Table 1, together with data as to depth of cutoff and drainage.

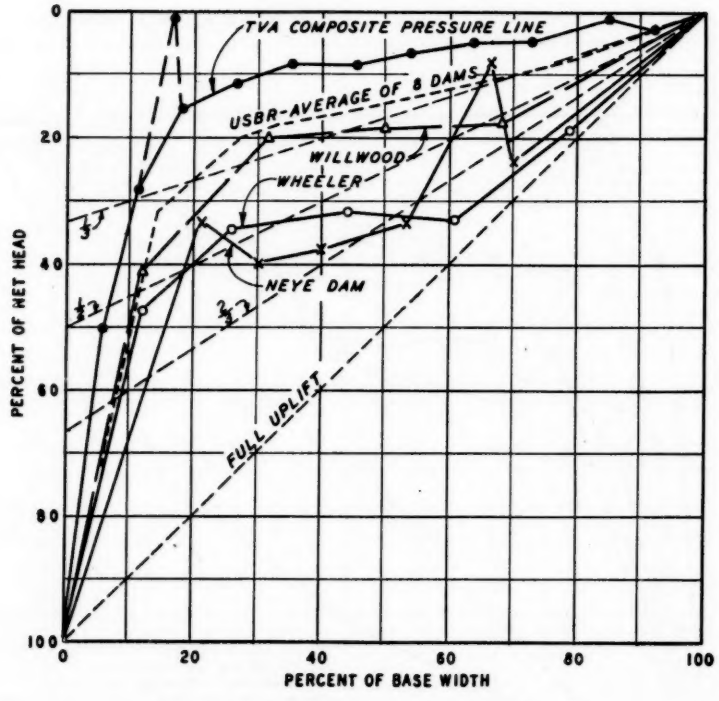


FIG. 4.—UPLIFT INTENSITY FOR UNDRAINED DAMS

Certain features of Fig. 5 require further explanation. It shows an average figure for Hoover Dam, extending over the period from 1935 to 1948. However, in about 1939 additional grouting and drainage were performed at Hoover Dam. Subsequent to this treatment the intensity diagram has been measured to have the form shown in Fig. 6. Obviously the additional work had a pronounced effect. The form of the intensity diagram for Owyhee Dam in Oregon is explained in Section 7*l*.

It is to be noted with reference to Mr. Keener's average curve that, if the later values for Hoover Dam had been used as obtained after the supple-

mentary drilling and grouting, and if Owyhee Dam with its rather exceptional foundation condition had been omitted, the average would have been closer to the TVA composite curve, referred to later, than appears in Fig. 5.

The uplift diagrams for Grand Coulee (in Washington) and Shasta dams are shown in Figs. 7 and 8,[17] respectively, and indicate a high degree of effectiveness in the drainage system. The Grand Coulee exhibit is given because it

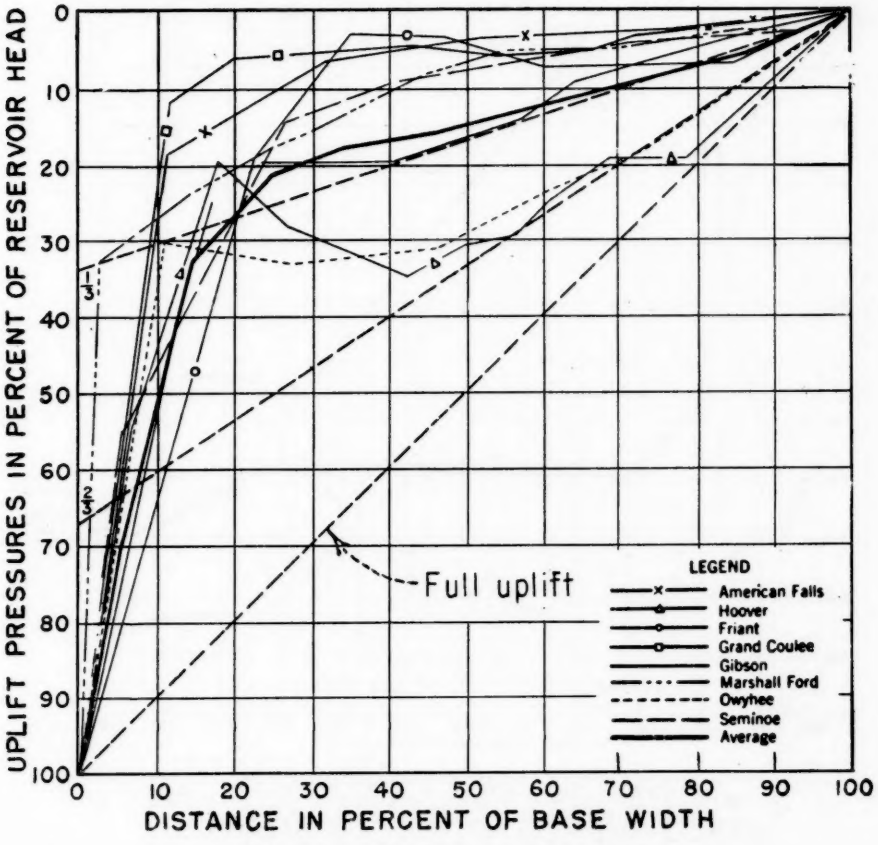
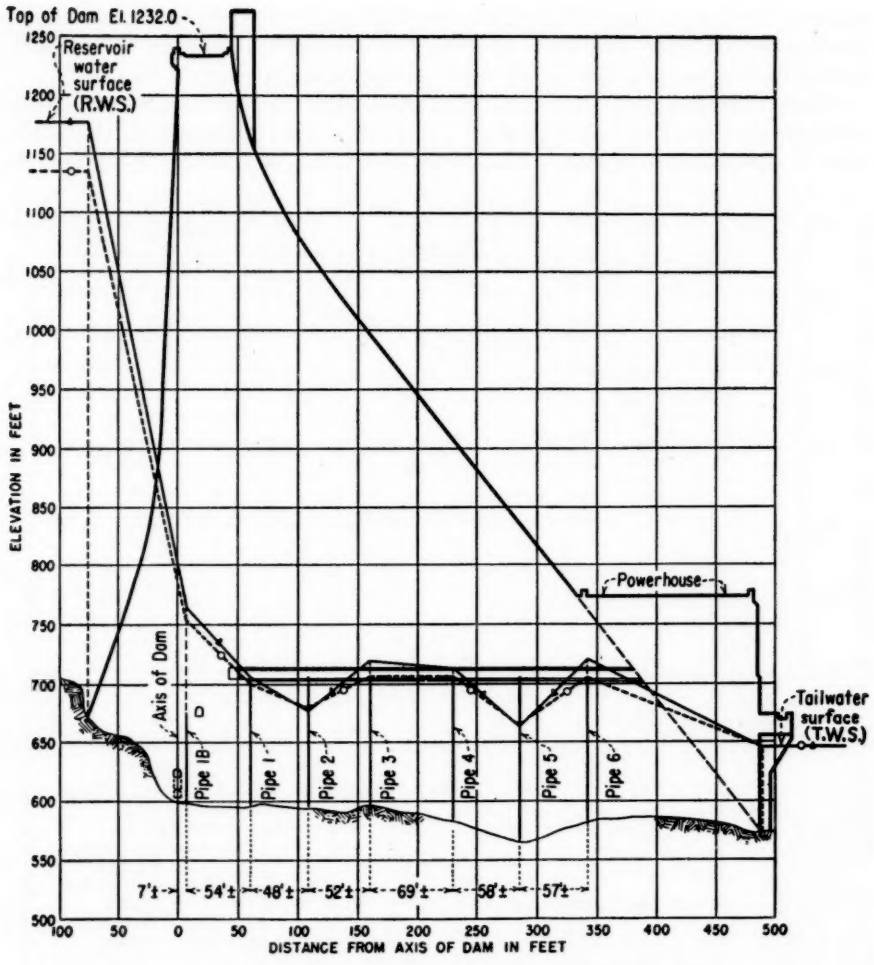


FIG. 5.—AVERAGE UPLIFT PRESSURE AT BASE OF SEVERAL DAMS

seems to represent the lowest intensity condition for the eight listed dams. The Shasta diagram is not included in Mr. Keener's general average. As to the Shasta diagram, Mr. Keener advises that the foundation rock is a solid and massive meta-andesite, very hard, with few and tight joints, although there are some crushed zones. The grout curtain is 150 ft deep at the maximum section with drain holes 10 ft on center extending 50 ft into the rock at the maximum section and 10 ft at the abutments.

[17] "Uplift Pressures in Concrete Dams," by Kenneth B. Keener, *Transactions*, ASCE, Vol. 116, 1951, p. 1227.



DATE	R.W.S. EL.	EL. OR EQUIVALENT EL. OF W.S. IN PIPE							T.W.S. EL.	LEGEND
		1B	1	2	3	4	5	6		
JAN. 15, 1947	1146.22	754	700	682	707	706	666	707	647.00	
FEB. 14,	1140.40	754	701	682	707	707	665	707	647.00	
MAR. 14,	1136.08	755	700	681	707	707	665	707	647.00	
APR. 15, 1947	1135.12	755	700	681	707	707	665	707	647.00	--○--
MAY 15,	1141.03	755	700	681	707	707	665	707	647.00	
JUNE 16,	1160.52	758	702	680	707	707	664	707	647.00	
JULY 15,	1175.65	762	704	680	707	707	664	707	647.00	
AUG. 15,	1177.23	766	706	680	719	714	665	721	647.00	
SEPT. 15,	1179.40	766	707	680	720	714	665	722	647.00	
OCT. 15,	1176.78	767	707	680	720	714	664	722	647.00	
NOV. 14, 1947	1177.01	766	707	679	720	714	664	722	647.00	—△—
DEC. 15, 1947	1173.55	766	707	679	720	714	664	722	647.00	

LINE B ALONG 10-FOOT GALLERY FROM ELEVATOR TO POWERHOUSE-NEVADA SIDE

FIG. 6.—HOOVER DAM (IN ARIZONA-NEVADA)—UPLIFT PRESSURES AT FOUNDATION; MEASUREMENTS DURING 1947

In Figs. 5 and 7 the uplift pressures are stated in terms of "reservoir head" rather than "net head." For these dams the effect of tailwater is not very significant, and the inaccuracy is not very important. The same can be said of Fig. 11 later.

e. In Fig. 9 is shown a composite of intensity profiles on four dams of the TVA.[18] In these dams observation sections were established as shown in

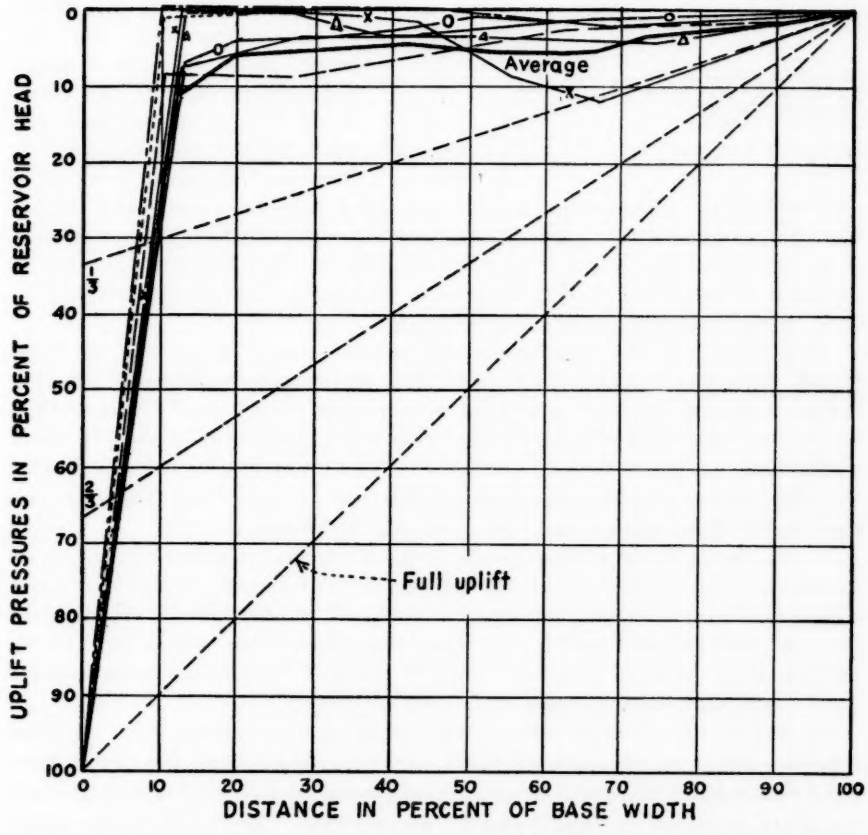


FIG. 7.—GRAND COULEE DAM (IN WASHINGTON)—UPLIFT PRESSURE AT BASE OF DAM, 1942-1945, ALONG VARIOUS LINES NORMAL TO AXIS. (HEAVY LINE IS THE AVERAGE)

Table 2. The intensity profiles used in constructing the composite were the maximum ones in the most recent year. Data on the character of foundation rock and on the drainage and cutoff provisions are given in Table 1(*b*).

The solid line as shown in Fig. 9 represents composite readings of intensity of pressure actually made in the area of contact between concrete and foundation or within the top 2 ft to 3 ft of foundation. Dotted lines are shown from

[18] Discussion by Ross M. Riegel of "Uplift Pressures in Concrete Dams," by Kenneth B. Keener *Transactions*, ASCE, Vol. 116, 1951, p. 1239.

the composite pressure line to a point representing the mean position of drainage outlets in a gallery. In the four dams there is perceptible flow from about 90% of the holes and in the remainder water stands at some level below the gallery floor. Although the drain holes do not fall exactly on the 14 measuring sections, there is justification for the view that the pressure gradient in the composite is no higher than the outlet of the drains in the gallery floor, and the dotted lines represent a gradient corresponding to this view. While there is considerable variation in the measurements as between individual sections of the total of 14, in no section is any measured pressure importantly higher than the assumed design gradient, as checked by the effect on the overturning moment. (This statement is on the authority of the Chairman, member of the staff of the TVA.) This diagram is furnished through the courtesy of C. E. Blee, M. ASCE, chief engineer, TVA.

TABLE 2.—DATA ON FOUR TVA DAMS

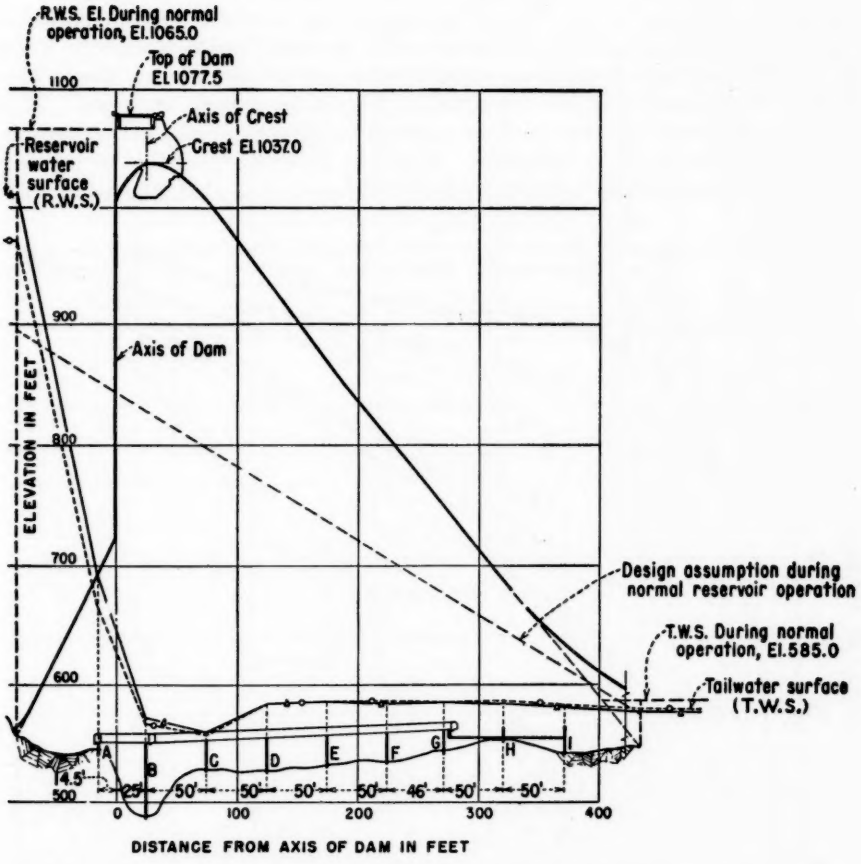
Dam	Maximum height, in feet	Number of sections observed
Hiwassee in North Carolina	307	4
Fontana in North Carolina	480	3
Douglas in Tennessee	202	3
Cherokee in Tennessee	175	4

One reason for the difference between the average curves for the eight USBR dams and the four TVA dams (Fig. 9) has been explained. Another is that the drain holes are somewhat more closely spaced on TVA dams than on most of the USBR dams. On the other hand, discussion by the Chairman of this Subcommittee with the Bureau of Reclamation engineers has indicated that the accuracy of Bureau measurements has probably been superior, and that the TVA intensities may have been slightly under-read.

f. At Norris Dam (in Tennessee) of the TVA, drains were provided at an average spacing of 34 ft. (This statement is on the authority of the Chairman.) This dam does not classify as either well drained or undrained. As might be expected, the uplift intensities are greater than those of the composite in Fig. 5, and less than those at Wheeler. Both Norris and Wheeler dams are founded upon calcareous rocks with nearly horizontal bedding.

g. As to dams provided with neither grouting nor drainage there are cited only two instances of measured uplift intensity, for example, the Oester Dam in Germany and Holtwood Dam on the Susquehanna River. Data as to measurements are given in Figs. 3 and 11, respectively. As to foundation conditions at Oester, little is known, but those at Holtwood are described in Section 7*m*. There is so little resemblance in these records that no important conclusions can be drawn.

h. Table 1(*c*) shows the characteristics of foundations and similar information to that in Tables 1(*a*) and 1(*b*) for a large number of dams constructed by the Corps of Engineers, United States Army. Unfortunately, uplift intensity measurements do not appear to be available to accompany this information. It is evident that the three major public organizations engaged in the construction of dams practice the systematic drainage of foundations, at least in recent years, and presumably have faith in its usefulness.



DATE	R.W.S. EL.	EL. OR EQUIVALENT EL. OF W.S. IN PIPE									T.W.S. EL.	LEGEND
		A	B	C	D	E	F	G	H	I		
JAN. 22, 1947	964.0	668	569	559	583	586	585	586	586	583	580.9	
FEB. 19,	974.1	673	569	559	585	587	586	587	586	583	577.8	
MAR. 18,	995.2	680	571	560	587	587	586	586	586	582	577.8	
APR. 16, 1947	1010.6	689	572	560	584	586	585	586	585	582	577.8	—o—
MAY 28,	1006.1	690	572	560	584	587	586	586	586	583	581.4	
JUNE 25,	1006.4	689	572	560	584	587	587	587	586	584	581.8	
JULY 23,	996.4	680	571	560	584	586	585	586	586	583	582.1	
AUG. 20,	984.9	673	570	559	583	586	586	586	586	583	581.7	
SEPT. 17,	976.1	668	569	559	582	585	585	585	585	583	579.8	
OCT. 15, 1947	972.3	665	568	559	584	586	586	586	586	583	579.7	----o----
NOV. 26,	973.1	664	569	559	583	586	583	585	585	582	579.3	
DEC. 23, 1947	972.8	664	569	559	583	585	585	586	585	582	578.2	

LINE 4 - STA. 26+06.75 ON ℄ OF TRANSVERSE GALLERY

FIG. 8.—SHASTA DAM (IN CALIFORNIA)—UPLIFT PRESSURES AT FOUNDATION; MEASUREMENTS DURING 1947

i. In connection with drainage, it may be noted that in Fontana Dam (in North Carolina) of the TVA a portion of the dam was provided with a secondary drainage gallery and drain holes at about the downstream third point, because the foundation conditions appeared to require this. The Bureau of Reclamation has also used a secondary gallery at times. This arrangement may sometimes be advantageous in other proposed structures if the geological study so indicates. It is also generally advisable to provide drainage under spillway aprons. When drainage is provided, the possibility

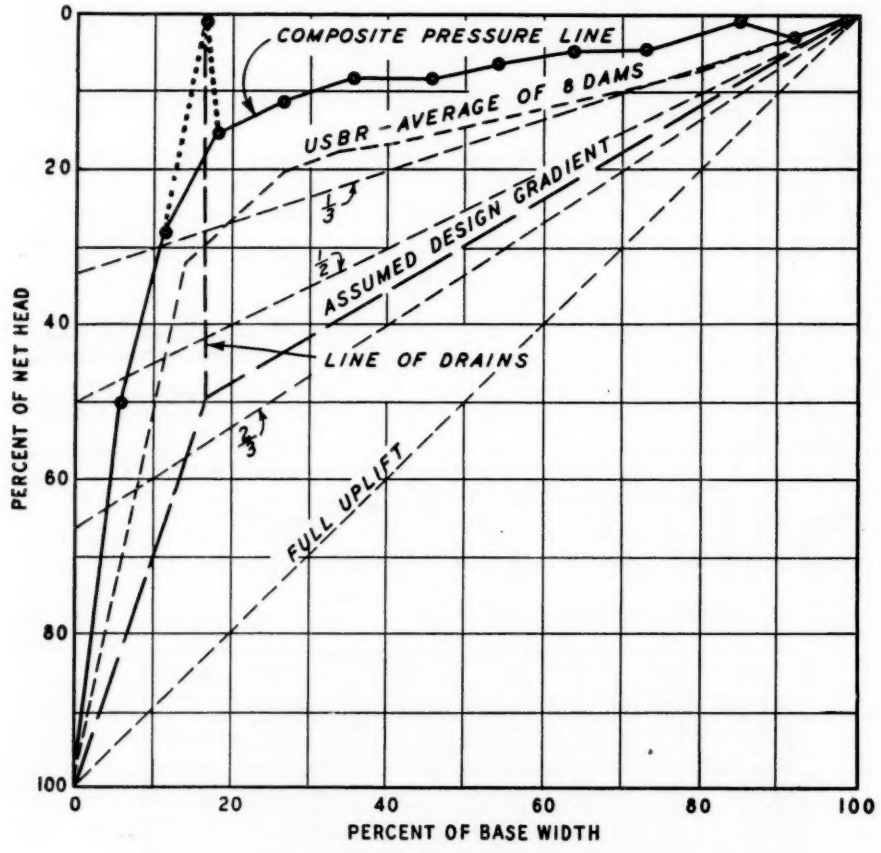


Fig. 9.—Composite Foundation Uplift Intensity for Four TVA Dams

of the drains becoming ineffective through deposits on the surface of the holes or in pores adjoining the holes must be recognized when active waters are circulating through the rocks. Regular inspection of the drained dams is a necessity.

When dams are constructed without drains but when grouted cutoffs are relied upon to reduce uplift (as would be justified by the three pertinent cases cited), a gallery should be provided in high dams in order to facilitate the

drilling of additional grout holes, if evidence should arise that the curtain is losing its effectiveness. Such evidence would be provided by an observed increase in leakage, or by an observed increase in uplift pressure. In low dams such grouting could be done through the water of the reservoir, but, if reservoir depths are much more than 50 ft, the difficulties of drilling are likely to be considerable.

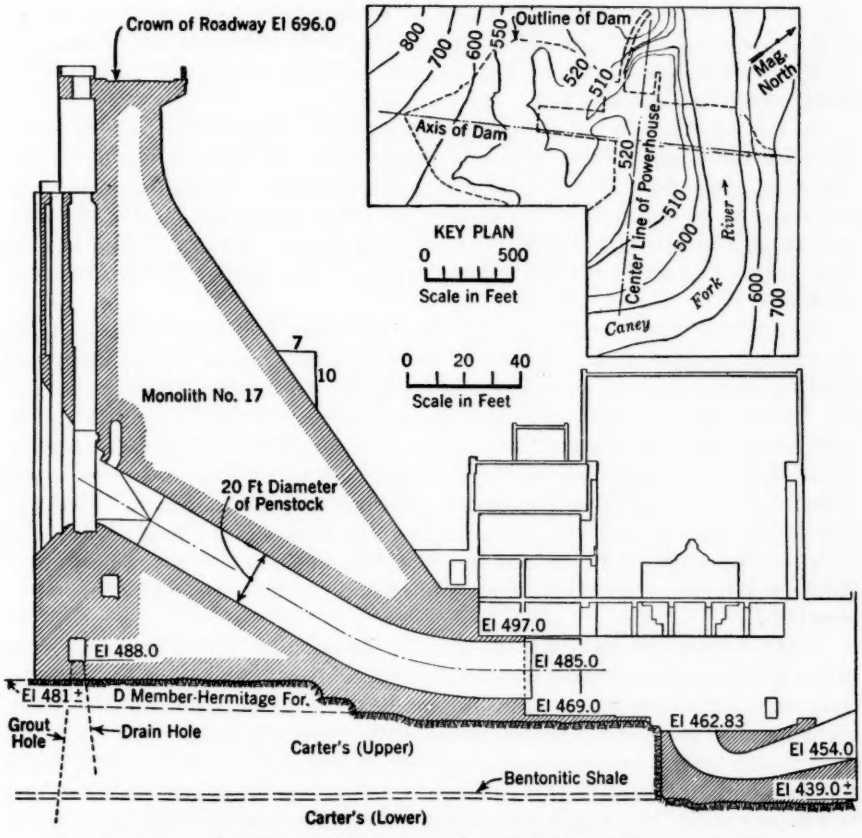


FIG. 10.—CENTER HILL DAM (IN TENNESSEE)—SECTION THROUGH DAM AND POWERHOUSE

In addition to the tabular and diagrammatic data presented, certain specific cases of foundation uplift phenomena will be discussed (Sections 7*j* to 7*o*) to show how exceptional conditions may affect the general question, or to supply data outside the scope of the tabular information.

j. Center Hill Dam (Tennessee).—This dam on Caney Fork, a tributary of the Cumberland River, about 50 miles east of Nashville, Tenn., is a concrete gravity and earth-fill type for flood control, navigation, and the generation of power—the cross section of the powerhouse section being shown in Fig 10. The dam itself was completed in December, 1949, including some foundation excavation for the powerhouse.

The concrete dam and powerhouse are founded on nearly horizontally bedded limestone containing certain impervious members. The strata dip moderately downstream and toward the right abutment (that is, approximately due north) so that the impervious members are exposed in the reservoir upstream from the dam but not in the river valley downstream from the dam. The foundation area thus overlies an artesian system several miles long.

Specifically, a layer of bentonitic shale, approximately 18 in. thick, was found in the exploration, lying about 40 ft below the base of the dam and 25 ft below the base of most of the powerhouse. It was penetrated, however, by the draft tube excavation (Fig. 10), the foundation concrete at this point being placed immediately below the bentonite bed.

In Fig. 10 it will be noted that there is a grouting gallery about 10 ft above the foundation and closely adjacent to the upstream face of the dam; this gallery is continuous through the concrete structure. Curtain grouting was performed from this gallery for a depth of 100 ft below the base of the dam and approximately 35 ft below the bentonitic shale. Relief drain holes were then drilled to a depth of 35 ft below the base of the dam and 10 ft above the bentonite.

During the winter and spring of 1948 and 1949, and prior to placing any appreciable weight of concrete in the draft tube excavation, the reservoir was filled to an elevation about 149 ft above the base of the dam and 189 ft above the bottom of the draft tube excavation. It was noted during this period that there was some upward movement of the rock in the zone of deep excavation for the draft tube, and a system of checks was set up to follow this movement and determine the cause. During the late spring the reservoir was drawn down to a level approximately 100 ft above the base of the dam when it was noted that the zone in which there had been upward movement then settled with reasonably close relationship to the lower elevation of the reservoir.

After further checks it was decided in the ensuing fall to drill test holes under the dam gallery and powerhouse area to discover whether or not there were any water bearing or pressure zones in the foundation rock at lower elevations. Two water pressure zones were developed under the dam and powerhouse area at levels of approximately 50 ft and 60 ft, respectively, under the dam and 10 ft and 20 ft, respectively, under the bentonitic shale. These test holes initially had a gage pressure at top of the hole of 47 lb per sq in. in the dam gallery upstream from the power intake section, and 42 lb per sq in. at top of the hole in the powerhouse area where upward movement had been noted.

It was then decided to drill additional drain holes on 20-ft centers in the dam gallery through the bentonitic shale to a level 20 ft above the bottom of the grout curtain. In addition, about 30 holes were drilled in the powerhouse area, below the powerhouse, along the channel side of the powerhouse, and at the downstream edge of the spillway bucket. The holes first drilled initially had high flows and pressures; but both flow and pressure were rapidly reduced with lapse of time and as other holes were drilled until all flow from holes in the dam gallery ceased and flow in the powerhouse area holes was

reduced to a negligible amount for the usual reservoir levels. It should be noted that the foundation rock, which had risen, had settled back again.

It was the opinion of consultants that the upward movement resulted from reservoir pressures superimposed on the artesian system and that, on relief of these artesian pressures, the drainage from the permanent relief holes would be negligible. It is to be noted that no water bearing or pressure levels were encountered in the concrete or in the foundation rock above the bentonitic shale. The Center Hill example illustrates the importance of geological study.

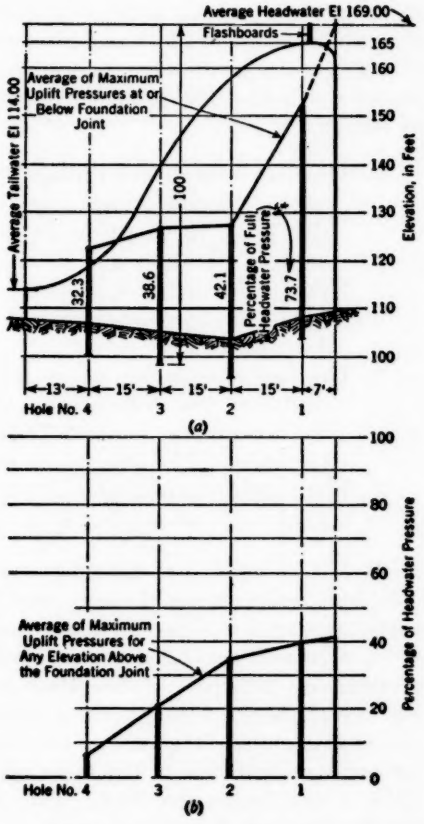


FIG. 11.—HOLTWOOD DAM (IN PENNSYLVANIA) —(*a*) AT FOUNDATION JOINT (*b*) ABOVE FOUNDATION JOINT

k. Chickamauga Dam (Tennessee).—At the Chickamauga Dam of the TVA there were several beds of bentonite in the limestone formations constituting the foundation of the dam. These beds dipped upstream but were broken somewhat by minor faults not far above the dam. The bentonite beds were 6 in. or less in thickness. During exploration artesian flows in drill holes were observed when the holes penetrated the bentonite beds, in a manner similar to that developed at Center Hill.

The bentonite also provided a floor against solution effects and in this respect was very useful from the standpoint of construction. A system of drain holes downstream of the grouted cutoff, penetrating the bentonite seams, was designed to minimize uplift intensity under the structures. The importance of geological relationships is again illustrated.

l. Owyhee Dam (Oregon).—With regard to the uplift diagram for Owyhee Dam (Fig. 5), Mr. Keener has furnished the following comment:

> "The uplift pressures at Owyhee Dam may be ascribed to the nature of the foundation rock, a porphyritic or glassy rhyolite, full of joints and fractures which are nearly vertical. The innumerable joints and fissures formed a widespread system of channels through which water could readily travel. Thus the drains were relatively effective only a short distance downstream from their locations as may be surmised from the shape of the diagram."

m. Holtwood Dam (Pennsylvania).—This dam (Fig. 11[19]), on the Susquehanna River, is an overflow ogee spillway about 70 ft in height above the lowest

[19] "Uplift Measurements at Holtwood Dam," by P. E. Gisiger, *Civil Engineering*, July, 1938, p. 447, Fig. 1.

foundation. The underlying rock is a metamorphosed granitic schist containing many quartz veins and occasional fault seams. No grouting or drainage was provided. The design assumed that pressure would vary from 50% of headwater to 50% of tailwater, applied on the full area ($A = 1.0$). The dam was constructed from 1906 to 1910.

In 1936, seventeen vertical holes were drilled through the dam, of which thirteen penetrated the foundation rock. Observations of pressures occurring at various levels were determined, making use of packers to isolate 5-ft sections of hole. The principal conclusions stated in the reference paper are as follows:

1. The four holes drilled through to rock next to the upstream face of the dam all showed substantial amounts of uplift at or near the foundation joint, varying from 51% to 84.5% of full head of forebay water at joint elevation, and averaging 73.7%.
2. The average maxima of uplift measured at or below the foundation joints, in percentage of full headwater pressure, are as shown in Fig 11(*a*).
3. The maximum uplift measured (in any of the four holes next to the upstream face drilled through to rock) for any horizontal joint above the foundation joint is 72.7% of full headwater pressure at joint elevation. The average of the four holes, however, is only 39.9%.
4. The average maxima of uplift measured for planes above the foundation joint are as shown in Fig 11(*b*).

It will be noted that measurements were also made in the concrete of the dam, this being almost the only case found where intensity of uplift was actually measured in these circumstances.

n. Stewartville and Aguasabon Dams (Ontario, Canada).—In "The Review of Research Activities of The Hydro-Electric Power Commission of Ontario for 1949," there are diagrams of observed foundation uplift for the Stewartville and Aguasabon dams of that organization (Fig. 12). The apparatus for measurement was of the same type as that used by the USBR and the TVA, as previously described. The observation points were in the inspection galleries, indicated on the diagrams.

These dams differ from those previously discussed in that in both cases there is a continuous surface drain at the rock surface parallel to the axis of the dam. Correspondence with the director of engineering of the commission, J. R. Montague, has furnished the following information:

Uplift measuring facilities have been provided for a number of the commission's more recent structures including the dams at Stewartville, Aguasabon, Des Joachims, LaCave, Pine Portage, and Tunnel projects, all in Ontario. The conditions at Stewartville and Aguasabon may be representative of the group. The rock underlying the Stewartville Dam is primarily crystalline limestone. Over a wide section of the foundation, steeply inclined faulting and subsequent solution along the fault zones have greatly altered the rock. As insurance against leakage a grout curtain was carried to a maximum depth

of 140 ft adjacent to the upstream face of the dam, which, in places, exceeded 200 ft in height with a net hydrostatic head of approximately 150 ft.

The rock under the Aguasabon Dam is mainly competent red granite. Here grouting was carried out along the face of the dam to a depth of approximately 70 ft. The maximum height of the dam itself was 120 ft. Most of the leakage here, which was of small proportions, originated through fine seams near the excavated rock surface and was readily gathered in the foundation drain.

The drain in both of these structures is continuous throughout the length, has a width of 15 in. and a minimum depth of 12 in. At Stewartville the drain leads directly to the powerhouse sump, the elevation of which is considerably

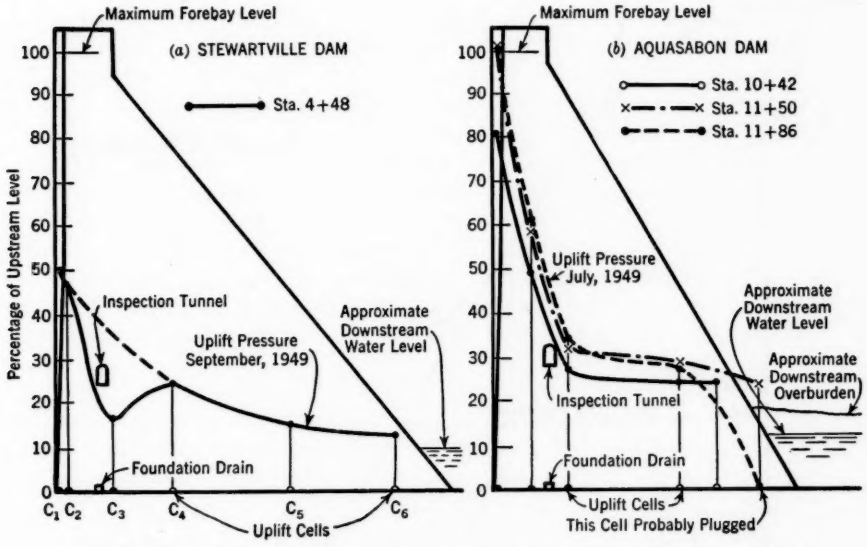


FIG. 12.—UPLIFT PRESSURE DISTRIBUTION FOR AGUASABON AND STEWARTVILLE DAMS IN ONTARIO, CANADA

lower than the tailrace. A by-pass to tailwater was also provided in case leakage should become excessive. At Aguasabon drainage is directly to the downstream side of the dam. In all cases the inspection tunnel and foundation drain are connected by a series of vertical 6-in. pipes spaced at from 20-ft to 38-ft centers. These provide emergency relief in case of blockage of the main drain and will facilitate drilling into the foundation rock should this be found desirable. Uplift along horizontal construction joints is obviated by the provision of drains leading to the inspection tunnel. Similar drains intercept leakage in the vertical construction joints.

It was considered that sufficient access to the drain was provided by the fractures and joints in the rock and that relief would thus be provided against any excessive pressure which might build up back of the grout curtain. Mr. Montague further states that any structure which might be placed on hori-

zontally stratified rock would be provided with vertical wells to considerable depth to relieve uplift on the various strata affected.

o. Bartlett's Ferry Dam (Georgia).—In a paper before the Hydraulics Division, ASCE,[20] some interesting data on uplift in this dam on the Chatta-

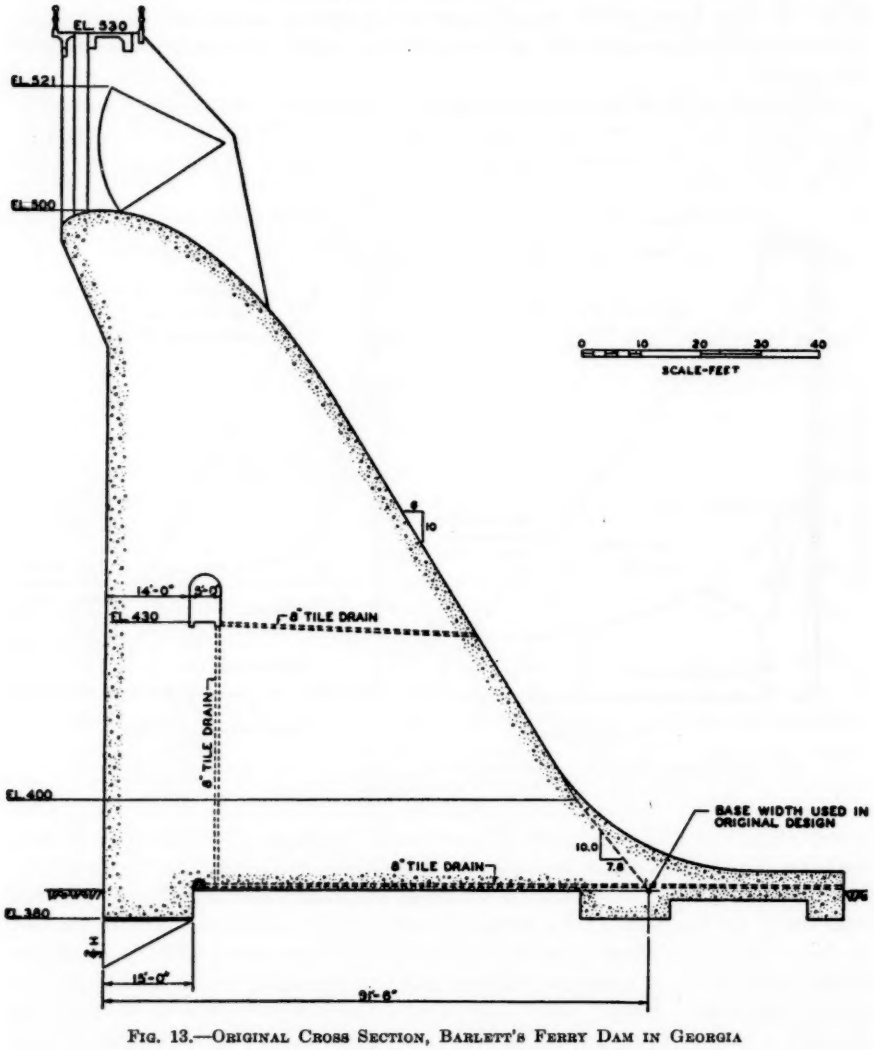


FIG. 13.—ORIGINAL CROSS SECTION, BARLETT'S FERRY DAM IN GEORGIA

hoochee River in Georgia are given. The following information has been taken from the paper.

This dam (Fig. 13) was built in 1925, and was acquired by purchase in 1929 by the Georgia Power Company, its present owner. The foundation

[20] "Remedial Modifications to 25-Year-Old Bartlett's Ferry Dam, Checked by Hydraulic Models," by E. S. Harrison and C. E. Kindsvater, *Proceedings-Separate* (ASCE, publication pending).

uplift assumption is shown in the figure. Mr. Harrison advises by letter that the assumed intensity was applied over 100% of the area, and that the longitudinal drain was assumed to prevent any uplift pressure downstream thereof.

The Georgia Power Company engineers came to believe that uplift pressures existed in the concrete above the rock. Accordingly, thirty-one 3-in. drain holes were drilled vertically at 20-ft centers. These were intended to reach the foundation, but in fifteen of the holes water, in such large quantities and pressures, was encountered that these holes had to be stopped temporarily. In eleven holes pressures of 100% of reservoir head were measured in the gallery while in four others over 90% was measured. Jets from the pipes rose to as much as 59 in. above the galley floor in one hole, from which the flow was approximately 390 gal per min.

The fifteen holes were then grouted, about 560 sacks of cement being used, after which the holes were redrilled and carried down into the foundation. In this condition, pressures ranging from 2% to 69% of reservoir head were measured in ten of the thirty-one holes although twenty-eight holes were recorded as having seeps or small flows. These conditions may be chiefly due to pressure intensity in the foundation. The pertinence of this information to the present discussion is that foundation uplift was in this case much less important than that in the concrete. The dam has been redesigned with the following assumption: Uplift pressure at any horizontal plane, to vary from 50% of full headwater to 0%, or tailwater, and to be applied to 100% of the base area. To secure satisfactory stability, a large buttress was constructed downstream of each spillway pier, and secured to the spillway structure.

8. Assumptions for Design

The observations of uplift intensity available to the Subcommittee have already been presented herein. The practices of the organizations reporting these data will now be stated.

a. The assumption made by the TVA engineers for the four drained dams cited is indicated in Fig. 9, in connection with the observations of intensity in the foundation. This assumption is based on an effective grout curtain, which will shut off the more permeable strata and, in case of uniformity in underlying formations, will have a depth equal to about one third of the head on the dam. Downstream of the cutoff at a distance from the heel about 10% to 15% of the width of base of the dam, drain holes are provided at a spacing of not more than 10 ft, and of a depth materially less than that of the grout curtain. In other words, the uplift assumption may be stated as follows: The intensity of pressure at the line of drains should exceed the maximum tailwater pressure by 50% of the difference between maximum headwater and maximum tailwater, the gradient then extending to headwater and tailwater, respectively, in straight lines. If there be no tailwater, as in the case of abutment sections of high dams, the downstream end of a similar diagram should be located at the surface of foundation rock. The area factor was assumed as two thirds, except for Fontana Dam where it was 1.0.

In further explanation of Fig. 9, the line designated "assumed design gradient" is not the real design gradient; Fig. 9 is based on the tailwater exist-

ing at the time when the field observations were made, and is drawn to show how the field observations compare with an uplift diagram of the type suggested. The true design diagram is based on the maximum tailwater and headwater to be expected under design flood conditions, for which condition, of course, there are no field observations. It is difficult to express the composite of the design flood conditions. The essential purpose of the diagram is to show the effectiveness of the drainage system.

A similar treatment of intensity has been favored by other engineers than those of the TVA.[21] The Owyhee Dam of the Bureau of Reclamation was designed on a similar intensity assumption, combined with the 100% area assumption. This treatment has the advantage that the gradient assumed for design is similar in form to the actual gradients noted in the TVA and USBR observations.

b. Inasmuch as the designer's judgment must determine the intensity factor, he may prefer, for simplicity, to assume the intensity factor I in terms of a straight-line variation, as shown in Fig. 3. There is no reasonable objection to such a procedure. For a discussion of some suggested values of this factor, reference may be made to a previously cited work.[5]

In Mr. Keener's paper[14] all the Bureau of Reclamation dams discussed are stated to be grouted and drained. The uplift intensity assumption assumed in design in all cases discussed (except Owyhee Dam) was that of a straight gradient from headwater to tailwater, herein designated as a 100% uplift, that is, $I = 1.00$ (Fig. 1). In all cases except Owyhee the area factor A was assumed at two thirds. This assumption was also used in the design of the Norris and Wheeler dams, referred to previously, these being designed for the TVA by the Bureau of Reclamation.

It appears in the light of the various figures presented that the 100% intensity factor ($I = 1$) is decidedly conservative, particularly if an area factor A of 1.00 be assumed, and that the intensity factor of the TVA diagram would be substantially supported by the USBR data.

c. The depth of the grouted cutoff near the heel of concrete dams designed by the Corps of Engineers is governed by foundation conditions that may affect seepage of water out of the reservoir. The depth of this cutoff bears no direct relation to the height of the dam. If impervious strata are available at reasonable depths, the cutoff grouting is carried to the impervious strata. Drain holes downstream from the grout holes are generally carried to about 80% of the depth of the grout holes. The grout and drain holes are generally drilled from a drainage gallery located in the concrete near the heel of the dam and a few feet off the foundation.

In the past the uplift was assumed as reservoir head at the upstream face, varying uniformly to tailwater head at the downstream face, applied to a percentage of the area of the base. Recent practice has assumed a percentage of reservoir head at the upstream face, varying uniformly to tailwater head at the downstream face, applied to the entire base area. Results obtained from these two uplift criteria are not materially different except in the case of

[21] Discussion by William P. Creager of "The Significance of Pore Pressure in Hydraulic Structures," by L. F. Harza, *Transactions*, ASCE, Vol. 114, 1949, p. 215.

unusually great tailwater depth, as noted in Section 3. (Statement of B. W. Steele, A. M. ASCE, member of the Subcommittee.)

d. If drainage be omitted and reliance for the reduction of uplift be placed only on the grouted cutoff, the available evidence is shown in Fig. 4. This shows a pattern of intensity favoring an assumption of less than 1.0 for the intensity factor *I*, but not so favorable as the USBR and TVA diagrams. Inasmuch as the undrained group comprise only three structures (as compared with twelve structures in the drained classification), a more conservative attitude toward this condition would seem to be required, that is, that a higher intensity factor should be assumed than would be assumed if drainage were provided in addition to cutoff. The designer's judgment must determine the proper assumption. A study of flow nets would be valuable in this connection. Those who would install drains but take no credit for their presence can still logically assume an intensity factor materially less than 1.0 if they use the grouted cutoff.

e. It may possibly be established by the geological exploration that the downstream portion of the dam rests on relatively tight strata whereas the upstream portion is founded upon relatively open structure. None of the examples cited herein is known to be in this category. For such a foundation, the uplift pressure is likely to equal reservoir pressure at the upstream face, to decrease slowly through the porous foundation, and to decrease rapidly through the tight portion. This would result in an uplift intensity greater than 1.0. Special foundation treatment methods might justify a reduction in the factor.

f. As to dams built without any cutoff grouting or drainage provisions, the following comments appear to be in order:

(1) It might appear that a 100% intensity factor should be applied. However, if foundation conditions are so unfavorable as to seem to justify such an assumption, they should be remedied by grouting, of the consolidation or the cutoff type, or both, by draining and by "dental work," and then an appropriate factor of intensity determined. These provisions apply chiefly to major structures.

(2) In minor structures it may be more economical to increase the intensity factor (and, consequently, the dam section) rather than undertake the cost of protective measures. By "minor structures" are meant those of less than, say, 100 ft in height. However, an intensity factor as high as 1.00 would seem to be only rarely justified.

g. Certain European engineers prefer to construct a gravity dam without drains or other openings which would provide shortened paths of seepage. They consistently employ an intensity factor of 1.0, even though the foundation be grouted. Such procedure is, of course, in accordance with their best judgment; but usually American engineers, including most of this Subcommittee, believe that considerations of economy generally dictate a different attitude.

h. Care should be taken to determine the maximum possible tailwater conditions, as they may be affected by ice jams or similar occurrences.

9. Design of Dam Above Foundation

The dimensions of the dam will be determined by the uplift assumption in the foundation. If drainage of the foundation be adopted, similar or equivalent provisions for relief of uplift or pore pressure may then properly be made in the dam itself. This may be done by wells similar in spacing to those in the foundation, by open contraction joints, by horizontal drainage of lift joints, or by other expedients, all designed to insure that no greater uplift effects occur in the dam structure than occur in the foundation. The same precautions as to location of drainage wells and ability to clean or supplement them should be observed as for the foundation.

The question of uplift in concrete is not covered in this report although some pertinent data thereon have been given. Certain investigations by the Corps of Engineers promise to provide an important contribution to the subject and to be fruitful material for any future committee study.

10. Conclusions and Recommendations

The value of this report lies chiefly in the data as to measurements and practice presented. Adequate design assumptions are the responsibility of the designer. However, a few conclusions or recommendations appear to be warranted, although they should not be taken as "codifying" design procedures.

a. The Subcommittee finds it inadvisable to recommend definite, or even minimum, values for the intensity factor I. The circumstances peculiar to each project must guide the designer in making the proper assumption. His own experience will be a factor in his judgment. The structural and geological characteristics of foundation rock are of fundamental importance, as evidenced by the case histories cited. The treatment of the foundation rock, as by cutoff grouting only, or by grouting combined with drainage, is equally important. The precedents and measurements cited in this report should be considered.

b. Most of the Subcommittee join in the conclusion that, for the design of new structures, it is advisable to assume an area factor of 1.0, for the reasons given in the body of the report.

c. The systematic inspection and maintenance of drainage systems are of great importance. In the opinion of this Subcommittee, if there is not adequate assurance that drains will be inspected and maintained, no credit for their performance should be assumed in design. Where the dams in question are in the custody of large organizations which maintain their own engineering staffs, it would appear that these organizations will automatically attend to such inspections and maintenance. Such are the USBR, the TVA, the Corps of Engineers, and the public utility systems which operate a number of hydroelectric projects. The projects operating under Federal Power Commission license receive such inspection from engineers of the commission. The Subcommittee believes that there is a duty upon any governmental authorities which may issue charters or licenses, or which may have the responsibility for approving plans for construction, to insure that such inspections are systematically made, either by the owners or by their own agents.

d. In a dam which is large enough to permit the installation it is advantageous to have an inspection gallery or galleries close to the foundation, whether

or not it be used for drainage. In this gallery it is advisable to install devices for measuring foundation uplift. Should unduly high uplift pressures develop, the gallery will facilitate their correction.

e. With the presentation of this report, the present Subcommittee should be considered as discharged. However, since the consideration of pore pressure or uplift in concrete has not been treated because of inadequate present information, it is recommended that a new committee or subcommittee be constituted to study and report upon this phase of the uplift question. Such a committee will probably have to function for several years before it will be able to report.

11. Need for Information

This report will not be satisfactory to those who are looking for finality in its conclusions. It cannot be made so in view of the lack of conclusive data in certain directions and in view of the fact that reliance for design assumptions must rest upon individual judgment. Necessary additional data include the following:

a. Additional observations of actual pore pressure intensity in concrete structures, including both those having provision for relief of such pressures and those lacking such provision.

b. Determinations of permeability of the concrete in such structures.

c. Similar determinations for foundation rocks, as and when feasible.

d. Determination of effective area in structures and foundations, if found possible.

e. Determinations of the intensity of uplift in existing dams and their foundations, particularly in those of considerable age. The methods employed at Holtwood are pertinent.

The accumulation of such data requires much time and considerable money, but it may be hoped that in the course of some years data will be accumulated. The public organizations, such as the USBR, the Corps of Engineers, and the TVA, have been trying, and may be expected to continue trying, to accumulate data. The owners of other dams, particularly those of light section, should be interested in answers to point "*e.*" As and when such information becomes available to an important extent, this report should be supplemented to incorporate the results. Meanwhile, the report represents the accomplishment of the Subcommittee to date.

Respectfully submitted,

John B. Alexander
William P. Creager
L. F. Harza
Ivan E. Houk
A. V. Karpov*
Gerard H. Matthes
Byram W. Steele*
James H. Stratton
Thomas H. Wiggin*

Ross M. Riegel, *Chairman*

Subcommittee on Uplift in Masonry Dams of the Committee on Masonry Dams of the Power Division

* Submitted individual comments, see Appendices. Mr. Wiggin did not sign the report.

Appendix A. Exception by A. V. Karpov

The writer believes the Subcommittee is not justified in specifying a 100% pressure area (conclusion 10*b*). In some conditions, where the rock stratification is horizontal or nearly so, the 100% area may be justified. When, however, good rock is encountered with vertical or close to vertical stratification, there is no justification for assuming a 100% area.

After all, the 100% area theory is based on pure speculation which, in the writer's opinion, is contrary to the available evidence. There is no use in increasing arbitrarily the cross section of a dam if that does not contribute to its safety.

Appendix B. Supplementary Statement by Byram W. Steele

The writer concurred in Section 10*b* because there is no other logical method of preparing a structural analysis of a section of a gravity dam except to assume 100% area and then apply to this area an intensity factor obtained from a thorough analysis of the geology and foundation conditions obtained at the dam site in question. Uplift is a force that must be recognized as a potential hazard in any water-retaining structure and hence, must be provided for in the design.

It is obvious that there will be differences of opinion among designers relative to the shape of the uplift intensity curve proposed for any design. It should be equally obvious in the case of Center Hill Dam cited herein that a curve in the form of a straight line between the headwater and the tailwater, as used so often in the past, would not have been adequate since the pressure actually developed was 100% of reservoir head over 100% of area. Foolproof drainage will cost only a fraction of what the assumption of full uplift costs.

To ignore drainage is to ignore an economic factor that is not in keeping with present-day engineering standards of design and economy. In addition to the system of drainage, designed and built as a part of the dam construction program, natural drainage through cracks and crevices in the rock and through construction joints in the concrete constitute important factors in molding the shape of the uplift curve.

Horizontally stratified rocks are potential sources of high uplift intensity unless relieved by proper and thorough drainage, whereas igneous rocks and columnar basalts provide a degree of drainage that is too often improperly evaluated in assuming an uplift value to use in the design of a dam with the result that the cost of the dam is excessive and in many cases the type of dam should have been changed from concrete gravity to one of the less expensive but equally acceptable types.

Appendix C. Minority Report by Thomas H. Wiggin

The report of the Subcommittee is a monument to the arduous labors, engineering knowledge, and experience of Chairman Riegel and it contains many valuable data and sound engineering opinion on uplift and related questions. Nevertheless, the writer, after much thought, refrained from signing the report because such signing would seem to indicate his belief that the

problem of uplift in masonry dams was now solved and the Society could forget it and pass on to other unsolved matters.

The writer was in favor of calling the report a progress report and of having the Subcommittee, after infusion of some younger blood, continue its studies and particularly extend them to the behavior of typical older dams which, with smaller base widths and without aid of grouted cutoffs or artificial drainage, continue to stand without apparent distress. If one's scientific conclusions appear not to square with human experience, it seems always safe to carry the research further.

Only three failures of masonry dams in the United States are generally mentioned as worthy of note and in each case foundation conditions were flagrantly bad and seemed sufficiently so to account for the failures. The dam at Austin (Pa.) which failed was not only on a poor foundation but had a base only 60% of its height.

The writer actively advocated the formation of an ASCE committee on uplift pressure and ice pressure by communication to the Society more than thirty years ago, again somewhat later when he was a member of the Society's Research Committee, and, finally, at the Chattanooga meeting in 1939 when he made two accepted motions for committees on uplift and ice pressure.

Lest the impression should be gained that the writer is in favor of skimpy dam sections, he would add that early in his technical schooling he noted what was apparently an inconsistency between factors of safety for framed structures and those for gravity masonry dams, and he has been responsible for increasing the factor of safety in a considerable number of older dams and is still frequently so engaged, sometimes by use of prestressed steel rods with specially designed anchorages. Provisions for factors of safety can never be wholly satisfactory, however, until the forces involved are more surely known.

It may be remarked, incidentally, that most dams in regions of cold winters would be rocked forward on their foundations or rocked at some upper joint if anything like the 30,000 or 40,000 lb per linear ft generally considered proper for ice pressure for monumental dams were actually developed. This statement may be considered as irrelevant to uplift pressure, but the writer has always associated the two problems of uplift and ice pressure and has long been disturbed that the national civil engineering society has in a century produced no complete and convincing answer as to the forces to be withstood by so important and potentially dangerous a type of structure as the gravity masonry dam. It may be added that the situation with respect to spillway provisions is in a similar state of uncertainty as between the provisions ordinarily made and the extraordinary provisions made in recent years where money was available.

The main conclusions of the report are that substantially 100% of the areas of foundation and joints receive uplift pressure, that the intensity of such pressure can be reduced materially below a straight pressure grade line from headwater level to tailwater level by grouting and drainage, but that such drainage should not be relied on unless systematic inspection at proper intervals over the life of the dam can be guaranteed. (It may be noted in passing that there is a possibility that pressures might be above the straight line, although

friction head from flow through a fine-grained porous medium, provided it is uniform, follows the first power.)

The fact is stated that Swedish engineers do not count on drainage. The wide buttress type of dam developed in Sweden with space between buttresses for drainage to the uncovered surface of the ground is noted. Some possible reasons are given for the continued stability of older dams designed and built without benefit of modern knowledge as to uplift pressures.

Apart from such corroboration as may be drawn from certain ingenious laboratory tests on small specimens subjected to combined compression and water pressure, proof of the existence of uplift pressure over nearly, if not quite, 100% of the base of masonry dams rests on the finding of pressure in dams of the USBR, TVA, and some others in practically all drill holes which cross the base joint. It is recognized in the noteworthy paper by Mr. Keener on tests of USBR dams that such drill holes cover only a microscopic proportion of the foundation area. At first glance, however, such tests would seem certainly to be equal in probability to a Gallup poll. The possibility seems to exist, however, that the drill holes, which extended 3 ft or more into the rock below the foundation joint, might pierce some minute passages in the rock not at the base joint. Furthermore, why should a base joint behave differently from a construction joint in the masonry on which experiments were also tried by USBR, as noted in the following paragraph.

As to the pressure at construction joints in concrete, Mr. Keener states that no such pressure was found. This would seem to be contrary to the conclusions of the experiments by Mr. Terzaghi, Leliavsky Bey, and others who found proof in the behavior of concrete laboratory specimens under stress that water pressure existed throughout the specimens. The USBR experiments on joints were given up after about 11 years without finding uplift pressures.

An interesting but unpublished paper (entitled "Hydrostatic Pressure in Concrete") by T. C. Powers, manager of basic research, Portland Cement Association, treats of the microstructure of concrete and evolves the conclusion that water permeates nearly but not quite the whole of concrete. However, he finds that the pressure is not disrupting but on the contrary is negative, except where the concrete lies below a continuously existing water level so that water can soak back into the concrete and keep its water-demand satisfied. As the joints under greatest stress are not unlikely to be below tailwater elevation, this theory does not afford much comfort with respect to the stability of the dam as a whole.

The report mentions the adhesion of mortar joints as one reason for the stability of light, ungrouted, and undrained dams. The writer recalls that years ago the late Howard A. Carson, Hon. M. ASCE, made many informative pioneer tests on unreinforced concrete beams 6 in. by 6 in. in section and about 6 ft long. A goodly number of these were further tested as follows: Halves of broken beams were put back in the forms and built out with fresh concrete to full length. A coating of neat cement was used at the joint. The beams were then retested with the joint in the middle and gave strengths nearly, and in

some cases quite, equal to that of the original 6-ft beams. The strength which these joints had would resist perhaps 400 ft of head without aid from weight.

Whatever the shortcomings of the old dam builders, most of them were careful with joints so that doubtless many of the older dams profit greatly by adhesion of joints.

In the repair and alteration of older dams the writer has sometimes seen strong stone break under the force of steel wedges, leaving the mortar intact; but the reverse is more often true and repointing and grouting are often necessary. Bottom joints however are seldom uncovered. They are better protected from the effects of leakage and do not suffer from frost action. Leaks in joints or elsewhere tend to enlarge by solution of the cement but fortunately the areas of joint affected are generally small in proportion to the whole, although sometimes a stalagmitic surface will be formed over the whole of the lower part of the downstream face of an older dam from well-distributed leakage higher up. Grouting cannot hope to restore strength of adhesion equal to the original strength of a well-made mortar or construction joint.

The writer is conscious of the inadequacy of this presentation, touching as it does on so few of the pertinent papers on the subject. He is, however, firmly of the opinion that the Society should press for further fundamental research as to uplift pressure, including investigations of typical older dams. Those who have conceived and carried out the tests considered in the Subcommittee's report are deserving of every commendation and Chairman Riegel likewise, not only for his share in the TVA tests but for his arduous labor in assembling the test data and putting so much information pertinent to uplift in form for study and reference.

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